Uncertainties in the design of support structures and foundations for offshore wind turbines

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A B S T R A C T

Offshore wind industry has exponentially grown in the last years. Despite this growth, there are still many uncertainties in this field. This paper analyzes some current uncertainties in the offshore wind market, with the aim of going one step further in the development of this sector. To do this, some already identified uncertainties compromising offshore wind farm structural design have been identified and described in the paper. Examples of these identified uncertainties are the design of the transition piece and the difficulties for the soil properties characterization.

Furthermore, this paper deals with other uncertainties not identified yet due to the limited experience in the sector. To do that, current and most used offshore wind standards and recommendations related to the design of foundation and support structures (IEC 61400-1, 2005; IEC 61400-3, 2009; DNV-OS-J101, Design of Offshore Wind Turbine, 2013 and Rules and Guidelines Germanischer Lloyd, WindEnergie, 2005) have been analyzed. These new identified uncertainties are related to the lifetime and return period, loads combination, scour phenomenon and its protection, Morison – Froude Krilov and diffraction regimes, wave theory, different scale and liquefaction.

In fact, there are a lot of improvements to make in this field. Some of them are mentioned in this paper, but the future experience in the matter will make it possible to detect more issues to be solved and improved.

1. Introduction

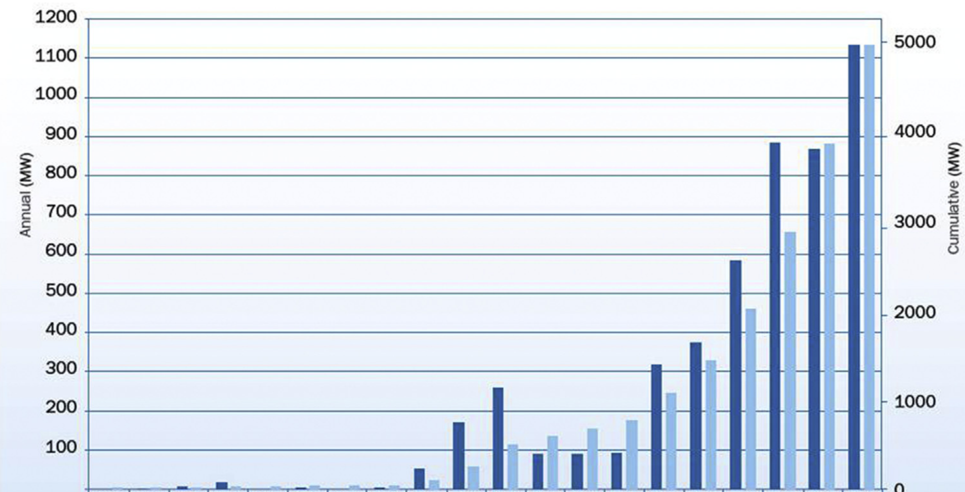
Offshore wind market has exponentially grown in the last years (see Fig. 1). According to statistical studies from the European Wind Energy Association (EWEA), 1166 megawatts (MWs) were installed in 2012, 33% more installed MWs than in 2011. Thus, there are currently 4995 MWs installed in 55 offshore wind farms in 10 European countries [1].

This growth, supported through different European Union Framework Programs, has its origin in the approach of ambitious challenges reached year after year in the last decade. Thus, the number of MWs installed has increased over time due to several factors such as the construction of more powerful wind turbine generators or the installation of a high number of wind turbines in each farm. These have been accomplished by greater depths, the improvement in foundation design and their scour protection systems, or the increased investment to connect wind farms to the grid.

However, despite this growth, there are still many uncertainties in the offshore wind energy industry [2]. These uncertainties can be due to a lack of experience. Some of these uncertainties are related to foundation design: loads combination and return period, scour phenomenon characterization under combined waves and currents actions, scour protection design, soil-structure interaction, wave loads determination, soil properties, transition piece design, different scales, etc.

This paper analyzes some current uncertainties in the offshore wind market, with the aim of going one step further in the development of this sector. To do this, some already identified uncertainties compromising offshore wind farm structural design have been identified and described in the paper. Furthermore, this paper deals with other uncertainties not identified yet due to the limited experience in the sector.

In fact, according to the study methodology conducted during the research, the paper begins identifying different requirements influencing the design of the foundations. Later, identified uncertainties for offshore wind foundation design are listed, and doubts and solutions are exposed according the current situation of the market. On the other hand, due to the limited experience in the



	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012
Annual	0	2	5	17	0	3	0	4	51	170	276	90	90	93	318	373	577	883	874	1,166
Cumulative	5	7	12	29	29	32	32	36	86	256	532	622	712	804	1,123	1,496	2,073	2,956	3,829	4,995

Country	UK	DK	BE	DE	NL	SE	FI	IE	NO	PT	Total
No. of farms	20	12	2	6	4	6	2	1	1	1	55
No. of turbines	870	416	91	68	124	75	9	7	1	1	1,662
Capacity installed (MW)	2,947.9	921	379.5	280.3	246.8	163.7	26.3	25.2	2.3	2	4,995

Fig. 1. Cumulative and annual offshore wind installations [1].

sector, some uncertainties have not been identified yet; these will be discussed in the paper with the aim of achieving an adequate and sustainable growth of the offshore wind technology.

2. Design requirements

The design of foundations and support structures of a wind turbine generator is very complex (Fig. 2 clarifies the meaning of "foundation" and "support structure" to be used along the paper). This implies taking into account numerous factors. Firstly, the different loads to consider for the structural design: wind turbine generator weight and loads due to the wind action, wave and current loads, operation and maintenance loads, etc. Also it is essential to consider terrain conditions and its main properties, construction and operation issues, and so on. The effect of all these issues, among others, makes the design of these structures very complex the design. However, there are some international recommendations and standards focused on this.

In force and current recommendations and standards for support structures and foundations design, with more relevance and use in the offshore wind industry, are the following ones:

- IEC 61400-1, 2005 [3].
- IEC 61400-3, 2009 [4].
- DNV-OS-J101, Design of Offshore Wind Turbine, 2013 [5]
- Guideline for the Certification of Offshore Wind Turbine, 2005 [6].

This paper is not intended as a critique of the before mentioned recommendations and standards, but some comments and contributions are given to help for improvements in the matter.

3. Existing uncertainties

Over the past 20 years, the rapid growth of offshore wind sector has been associated with the need to improve the design requirements present in offshore wind farms. To improve the design of these structures, it is necessary to know in depth the response of the foundations to the requests of external agents, their response to the fatigue during the operation phase, and the main characteristics of the seabed in which they are located. Therefore, nowadays there are still many uncertainties that question the design requirements used so far.

One of the most discussed uncertainties in the sector is the transition piece issue. The transition piece provides the connection between the support structure and the wind turbine generator. It represents the main weakness of the monopile foundation concept. The transition piece is jointed to the monopile using grouting to transfer all the loads and forces from the wind turbine tower through the transition piece down to the support structure.

Due to the wind and waves dynamic loads, grouting inside the transition piece crumbles (see Fig. 3). In many cases, there are not any clear solutions for this, but nowadays it is common to refill these pieces with new grout, to complete the connection with shear keys or to use conical instead of tubular sections (see Fig. 4).

On the other hand, soil condition is a key issue for the foundations design. A detailed knowledge of the nature and composition of the seabed remains a complicated and expensive task that requires a large investment in carrying out the design of foundations present in offshore wind farms.

In order to reduce costs, the characterization of the seabed in the area where a wind farm will be installed is usually done through a limited number of samples. Given the scarce number of samples taken, and assuming the non-homogeneity of the seabed in most

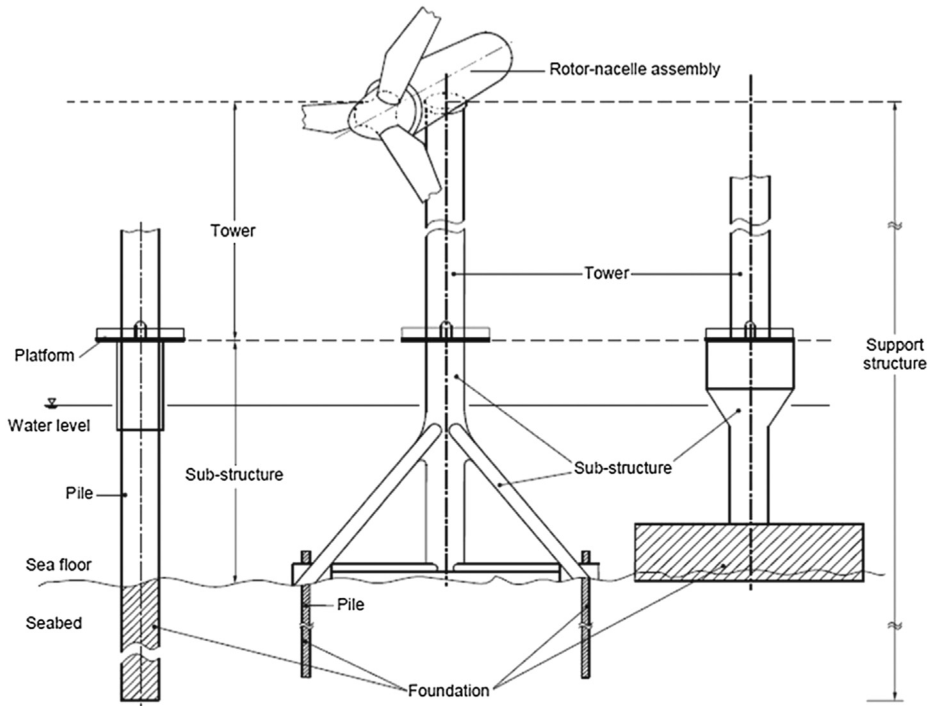


Fig. 2. Offshore wind turbine structure components [3].

cases, it is evident that the soil characterization remains some uncertainties although non-intrusive methods like geophysical campaigns are used complementary to the results from intrusive test like boreholes and CPTs.

Bundesamt für Seeschifffahrt und Hydrographie (BSH), from Germany, has written and published the Standard “Ground Investigation for Offshore Wind Farms” [8], giving some minimum recommendation for geological and geotechnical studies in order to achieve a suitable soil characterization in the offshore wind farm location.

4. New detected uncertainties

Main uncertainties already detected industry for the structural design of foundations and support structures in the offshore wind have been listed in the previous paragraph. Once analyzed the most used recommendations and standards, new uncertainties have been identified and discussed in next paragraphs.

4.1. Lifetime and return period

IEC standards [3,4] indicate a design lifetime for wind turbine generator to be at least 20 years. Possibly due to this fact, the minimum design service life for substructures and foundations for offshore wind turbines defined in these recommendations is also 20 years.

On the other hand, DNV [5] recommends 10^{-4} nominal annual probability of failure, related to a normal safety class. In case of manned structures, the nominal annual probability failure is 10^{-5}. Wind turbines foundations and support structures must be designed for the 10^{-4} value, corresponding to the case of unmanned structures, because operation and maintenance personnel will not be in the wind turbine structure location during severe

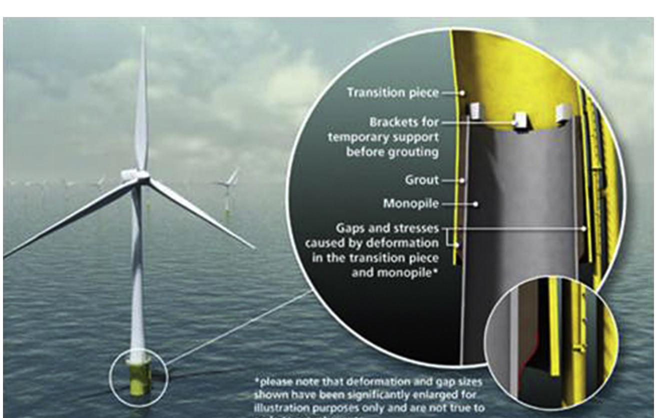


Fig. 3. Typical design of the transition piece [7].

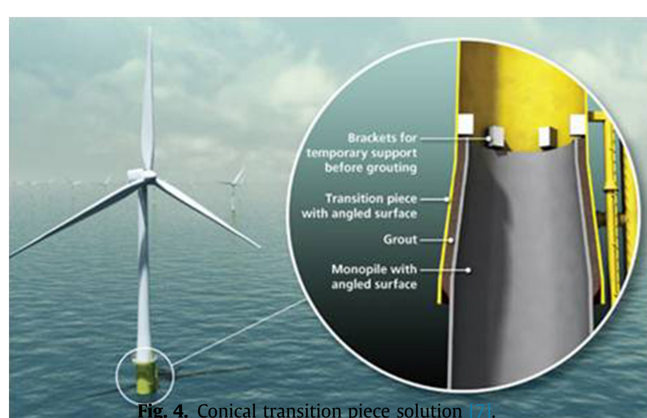


Fig. 4. Conical transition piece solution [7].

Table 1
Minimum lifetime depending on Economical Repercussion Index (low if $\mathrm{ERI} < 5$; moderate if $6 < \mathrm{ERI} < 20$; and high if $\mathrm{ERI} > 20$) [9].

ERI	≤5	6–20	>20
Useful life in years	15	25	50

Table 2
Failure probability depending on Social and Environmental Repercussion Index (no if $\mathrm{SERI} < 5$; low if $5 < \mathrm{SERI} < 19$; high if $20 < \mathrm{SERI} < 29$; and very high if $\mathrm{SERI} > 30$) [9].

SERI	<5	5–19	20–29	>20
P_{fELU}	0.20	0.10	0.01	0.0001
β_{ELU}	0.84	1.28	2.32	3.71

loading conditions. 10^{-5} value should be considered in case, for example, of offshore transformer substations foundations and support structures design for maintenance personnel living or staying there during severe loading conditions.

This statement has been analyzed considering equations and formulas from ROM 0.0 [9]. ROM is a Spanish standard program with several technical guides written and published by "Puertos del Estado" (Spanish Port Authority) focused on maritime engineering matters, mainly harbors issues. Using the following formula from ROM 0.0 "General Procedure and Requirements in the Design of Harbor and Maritime Structures":

$$T_{\mathrm{r}} = \frac{-n}{\mathrm{Ln}\left(1 - P_{\mathrm{f}}\right)}$$

where "n" is the design lifetime and P_{f} is the nominal annual probability of failure.

According to this formula, with $P_{\mathrm{f}} = 10^{-4}$ and $n = 20$ years, the return period to consider is around 199,990 years; with $P_{\mathrm{f}} = 10^{-5}$ and $n = 20$ years, the return period to consider is around 1,999,990 years. These values are huge related to the habitual ones employed for maritime structures design, and as it will be explained throughout this paper, it is not in line with the statements from the offshore wind recommendations regarding return periods. Therefore, this point is worth paying attention to.

DNV indicates that, for the ULS (ultimate limit state), characteristic loads are related to the 98% quantile, and therefore, to 50 years return period. A very different value to 199,990 years previously calculated. Issues related to load combinations must be treated, as it will be shown in the next paragraphs.

ROM 0.0 exposes a procedure to estimate the return period depending on the economic repercussion (low, moderate or high ERI o Economic Repercussion Index) and on the social and economical repercussion (SERI or Social and Environmental Repercussion Index, classified in non-existent, low, high and very high social and environmental impact) (Tables 1 and 2).

Table 4
Event probability (%) for various return periods (years) and various design life (years) [10].

Design life (years)	Event probability (per cent) for various return period (years)								
	5	10	20	30	50	100	200	500	1000
1	20	10	5	3	2	1	<1	<1	<1
2	36	19	10	7	4	2	1	<1	<1
3	49	27	14	10	6	3	1	<1	<1
5	67	41	23	16	10	5	2	1	<1
7	79	52	30	21	13	7	3	1	1
10	89	65	40	29	18	10	5	2	1
15	96	79	54	40	26	14	7	3	1
20	99	88	64	49	33	18	10	4	2
30	>99	96	78	64	**45**	26	**14**	6	3
50	>99	99	92	82	64	39	22	9	4
75	>99	>99	98	92	78	53	31	14	7
100	>99	>99	99	97	87	63	39	18	10
150	>99	>99	>99	99	95	78	53	26	14
200	>99	>99	>99	>99	98	87	63	33	18
300	>99	>99	>99	>99	>99	95	78	45	26
500	>99	>99	>99	>99	>99	99	87	63	39
1000	>99	>99	>99	>99	>99	>99	99	86	63

The following tables show the return period values resulting from the application of the standard [9], for different lifetime figures (15, 25 and 50 years), the interest of the works (local or general interest) and related to different maritime structures types (beaches, and their groins, sewage pipes, rubble mound breakwaters with gradual failure and vertical breakwaters with instantaneous failure) (Table 3).

In case of using Ref. [9] for the design of offshore wind farms structures, due to the instantaneous failure of the different support structures and foundation used for offshore wind turbines (monopiles, gravity based structures or GBS, tripods or jackets are the most common ones), 237 or 475 years of return period should be used, enough higher values than the 50 years one recommended by offshore wind standards. Considering the minor of them, i.e., 237 years, the difference is evident.

It is very interesting to analyze Table 4 [10]. Considering 20 years lifetime and 50 years return period, as offshore wind recommendations, it can be statically demonstrated that the probability of occurrence of the wave height associated to is about 33%, a high and surprising percentage, creating doubts about the consistency of the structural design based on these figures. On the other hand, considering Ref. [9] recommendation, 20 years lifetime and around 200 years return period, the probability of the event is 10%, and with 500 years return period, the probability is 4%.

4.2. Load combination

In the case of ULS [5], recommends to take the characteristic combined load effect as the 98% quantile in the distribution of the annual maximum combined loads, i.e. the combined load effects with 50 years of return period (Table 5).

Table 3
Return periods for different design lifetime using ROM 0.0.

Lifetime/interest	Failure probability	Exceedence probability	Return period/ recurrence ($1/P$)	Maritime structure type
15 years	0.20	0.06667	68 years	Beaches, and their groins
15 years	0.10	0.00699	142 years	Outfalls, pipes, pipelines
25 years	0.20	0.00886	112 years	Rubble mound breakwaters with gradual failure
25 years/local interest	0.10	0.00420	237 years	Vertical breakwaters with instantaneous failure
50 years/general interest	0.20	0.00452	225 years	Rubble mound breakwaters with gradual failure
50 years/general interest	0.10	0.00210	475 years	Vertical breakwaters with instantaneous failure

Table 5
Characteristic values of environmental loads or load effects, which are specified as the 98% quantile in the distribution of the annual maximum of the load or load effects, shall be estimated by their central estimates [5].

Statistical terms used for specification of characteristic loads and load effects			
Term	Return period (years)	Quantile in distribution of annual maximum	Probability of exceedance in distribution of annual maximum
100-year value	100	99% quantile	0.01
50-year value	50	98% quantile	0.02
10-year value	10	90% quantile	0.10
5-year value	5	80% quantile	0.20
1-year value	–	Most probable highest value in one year	

Accidental loads are essential for the structural design. While seismic ones are considered in offshore wind standards with 475 years of return period, wave actions are not consider as accidental loads. The existence of a similar paragraph in Ref. [5] and in Ref. [11] really attracts attention. This paragraph is about the use of 50 years of return period characteristic loads, but as it is exposed in Ref. [11], this should be the return period for permanent loads and variable functional loads due to operation and maintenance overloading; in the case of wave load, the 98% quantile corresponding to 50 years of return period must be considered; in addition to this [11], indicates that for extraordinary actions like seismic and extraordinary waves, the characteristic value of the action shall be that corresponding to a 500 years return period.

4.3. Scouring

The scour phenomenon (see Fig. 5) jeopardizes the operating capacity of offshore structures since it compromises their stability [12]. So far, different investigations have been carried out linked to the origin of the scour process and its development in bridge piers (generally under steady current conditions). The study of this phenomenon in the marine environment for different authors like [13] or [14], began a few years ago in the field of offshore wind farms, considering that these structures are jointly subjected to currents, tides and waves, in a different regime than bridge piers.

As is mentioned in Ref. [13], in the marine environment the time-varying nature of the waves and currents makes the problem more complex than that of scour at structures in rivers. Much research work carried out on scour phenomenon in offshore wind farms with monopile foundations has obtained different formulations and methods, that allow this phenomenon to be characterized by predicting maximum scour depth (S_{max}) and maximum scour extension (L_{ext}) in the vicinity of the pile. Different authors like [15] characterized the maximum scour depth under steady current conditions. Sumer [14] proposed a new formula to estimate this parameter only under the effect of wave, but until 2002 a new formula to predict maximum scour depth at equilibrium was not proposed.

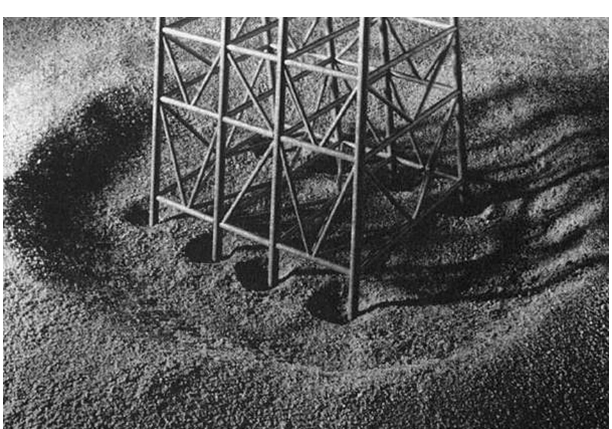


Fig. 5. Global and local scour development around a jacket structure [13].

The characterization of this phenomenon, knowing the serious consequences related to its occurring (loss of structural stability, sliding, etc.) has evidenced over the last few years, the need to develop methods and systems for the protection of these offshore structures. Scour protections are required to prevent problems of structural stability and may be required also to protect the inter-array and export cables.

Surprisingly, nowadays different offshore standards like [5] proposes the use of [16] formula for scour characterization around offshore wind turbines under the combined currents and waves actions, which is a great inaccuracy.

The design of scour protection shall be integrated into the foundations design. In order to carry out an effective design, sediment properties, seabed's geotechnical characteristics, environmental parameters (Hs – significant wave height, Tp – peak period, etc.), turbine specifications (diameter, shape of pile, etc.) have to be taken into account and must accurately predict the maximum scour that would occur in the absence of this protection.

Taking into account the design of scour protections, it would be advisable to size these structures using climatic variables and also depending on geotechnical properties of the terrain in the location [12] recommends to design scour protection with extensions between L/4 and L/2 (L is wave length). Furthermore, these structures have been studied according dimensionless wave height parameter ($H_0 = H_s/(\Delta D_{50})$), where H_s is the significant wave height, Δ is the relative mass density and D_{50} is the characteristic diameter of the natural material (gravel, stone or sand depending of the type of structure to be studied). As a consequence, scour protection systems have been classified with the dimensionless wave height parameter between 6 and 15 [12].

When physical models have been used up to now for the scour protection analysis, scale factors applied have not been the right ones. In fact, monopile diameter, scour protection stones, and seabed sand have been characterized using different geometry scale factors.

4.4. Morison, Froude–Krilov and diffraction regimes

It is essential to know if the structure to design is within Morison, Froude–Krilov or diffraction regimes. This is a key issue to estimate wave forces over the structure. Morison regime is analyzed in depth in offshore wind recommendations, with some formulas for drag and inertia loads estimations. Froude–Krilov is not analyzed in offshore wind recommendations previously listed. And the only reference to the diffraction in DNV is that this occurs when the structure modifies the wave pattern, i.e. when the cross sectional dimension of the structure is large compared to the wave length, typically when $D > 0.2\lambda$ (D is the main cross sectional dimension, and λ is the wave length), situation when Morison is not applicable. But no recommendation for the application of diffraction regime is given, being important above all when designing gravity based structures with a large cross dimensional section compared to the wave length.

Fig. 6 [5] represents the regimen conditions for the structure depending on H/D and λ/D values (where H is the wave height,

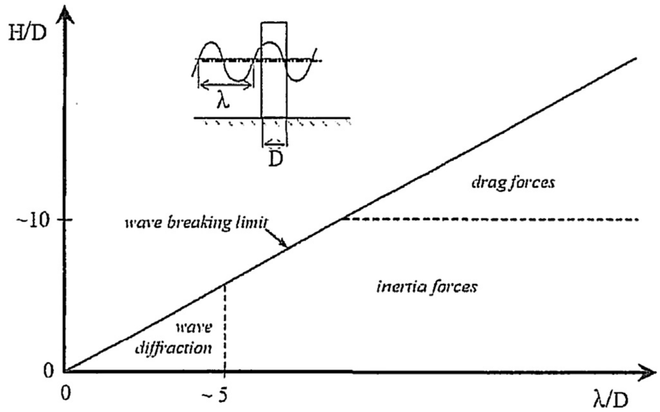


Fig. 6. Relative importance of inertia, drag and diffraction wave forces [5].

without indicating if it is significant or maximum or $H_{1/100}$ or other one; D the cross sectional dimension; and λ the wave length).

Other classifications exist to identify the regimen for estimation of wave forces, like the one created by Ref. [17]: Morison to be used when $D/L < 0.05$ (where $L = \lambda$, the wave length); Froude–Krilov when $0.05 < D/L < 0.20$; and diffraction when $D/L > 0.20$.

Another classification was made by Ref. [18]: Morison to be used when $D/L < 0.10$; Froude–Krilov when $0.10 < D/L < 0.20$; and diffraction when $D/L > 0.20$. A more sophisticated diagram [18,19] is shown in Fig. 7, with different regions depending on H/D and $\pi D/\lambda$, using maximum wave height and medium wave period: deep water breaking wave curve, all inertia (negligible drag and diffraction),

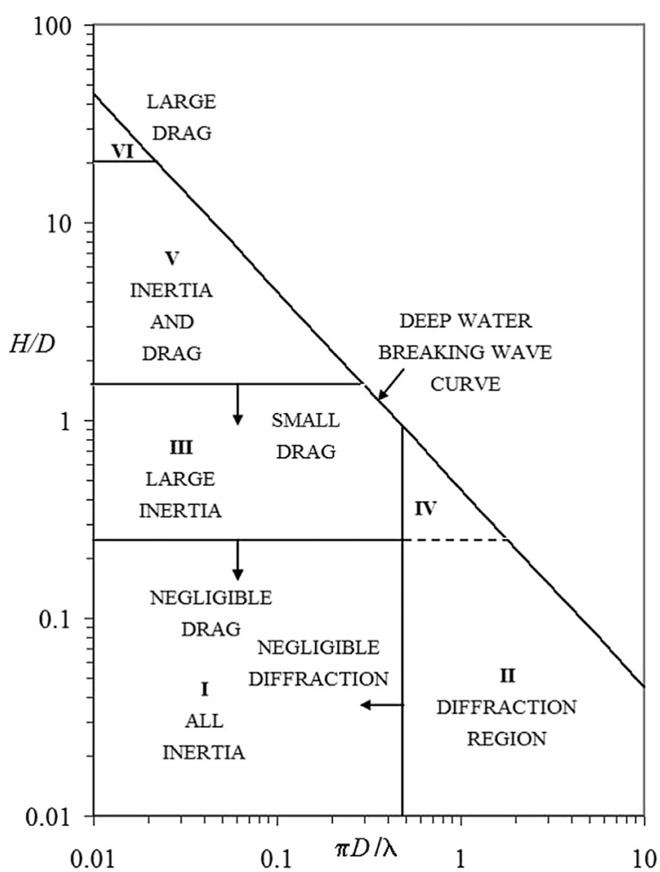


Fig. 7. Different wave force regimes [18].

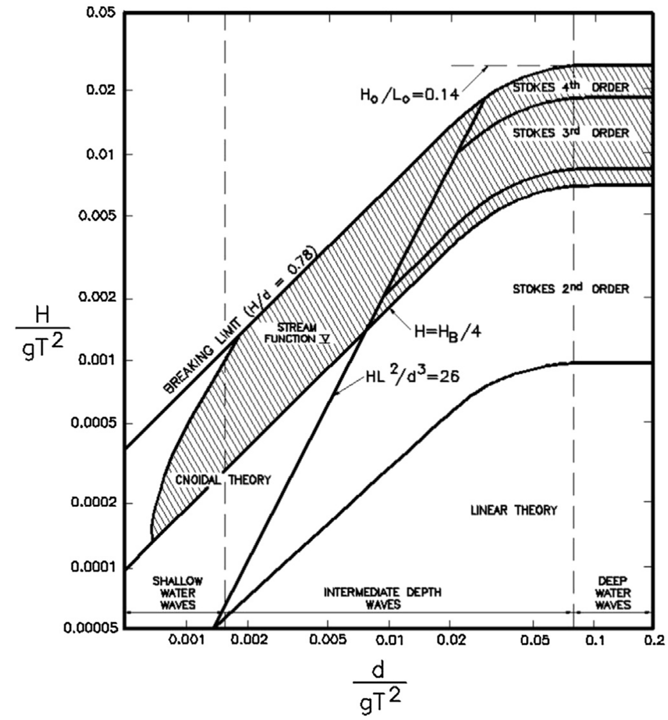


Fig. 8. Wave theories according to Lè Mèhautè [20].

diffraction region, large inertia (small drag), inertia and drag, and large drag.

As a result of these statements, it is not perfectly clear the identification of the regimen for wave forces estimation. Other important issue to be analyzed is if Ref. [18] formulation can be applied for big pile diameter (around 5 m), knowing that if $H_{max}/D < 2$ y KC < 6, inertia is dominant, and that if $H_{max}/D > 20$ y KC > 60, drag is dominant (V and VI regions in Fig. 7).

4.5. Wave theory

Other important issue is the wave regime to be considered for the estimation of wave forces, scouring, etc. The wave theory included generally on the equations is the lineal or Airy one.

Lineal theory is rare the most suitable wave theory. When general project data are introduced on Lè Mèhautè diagram (Fig. 8), the most usual theories are Stokes and Cnoidal. This can imply some uncertainties in the structural check.

Wave variables selection is important. The wave height assumed can be the significant wave height (H_s) or the maximum wave height (H_{max}). And the wave period is not the intrinsic one according DNV standard; the right one is the most stable in statistical or in spectral terms: the medium period (T_m or T_{02}) [19].

4.6. Different scale

Up to now, typical piles used in maritime engineering have a maximum diameter around 2 m. On the other hand, monopiles used in offshore wind facilities, have a diameter around 5 m or even bigger diameters. The different scale is evident, and this should be considered. In fact, some formulas used for monopile design are indicated for up to 2 m diameter piles; for example, finite element models have shown that the API p–y method overestimates soil-pile resistance [21]. This can be risky due to the different scale. Also it is important to consider the maximum pile diameter depending on the existing installation hammers and barges.

Changes in scale maritime engineering have been frequently associated with failures of components and/or systems. One example is the use of dolos like concrete armor units for breakwaters. Dolos were developed in South Africa in 1966 by Zwamborn and Merryfield with units between 6 and 12 tons size. 42 tons size dolos units were used in Sines breakwater in 1978, and 50 and 51 tons size dolos units were used in San Ciprian (Lugo, Spain) in 1979. It is evident the scale difference between Sines/San Ciprian and the original ones in South Africa. Wave storm events during February and March in 1978 implied the collapse of the Portuguese breakwater. And wave storm event during December in 1979 produced serious failures in the head of the San Ciprian north breakwater. These failures demonstrated that dolos units work in different way when the original scale is not respected. In fact, the scale factor was not taking into account. The diameter of monopiles used in offshore wind facilities is around 5 m, and growing; but the piles used up to now in port facilities are less than 3 m. The scale difference is noticeable.

4.7. Liquefaction

An important issue to be considered is the possible liquefaction in sandy terrains where pile foundations are used. It is necessary to analyze this issue in all the phases of the project, including construction and operation of the offshore wind facility.

DNV indicates that liquefaction has to be taken into account in seismic areas, but this has to be considered also during the pile driving and even during the operation phase because the dynamic loads.

5. Conclusions

The current development of offshore wind technology must imply a lot of improvements and progresses in different matters, one of them being the design of foundation and support structures for offshore wind turbine generators.

There are numerous aspects influencing this design, like meteocean, wind turbine generator and operation loads, terrain properties, and so on. It makes the structural design very complex.

Some uncertainties are well known, and in fact, these are in the process of being improved and solved, but the question is not easy at all. Examples of these uncertainties are the design of the transition piece and the difficulties for the soil properties characterization. The difficulties about the design of the transition piece are being studied, and different alternatives are being proposed: shear keys between the grout and the steel structures (monopile and transition piece), conical shape of the own transition piece, etc. The second one is due to the heterogeneous soil characteristics, and the risks related to it can be reduced with a huge investment and a time consuming soil campaign.

Current and most used offshore wind standards and recommendations related to the design of foundation and support structures ([3–6]) have been analyzed. The results of this analysis have been mentioned throughout this paper, being summarized as follows:

- Wave action is not considered as accidental load. Return period of 50 years for a lifetime of 20 years is habitually used for existing offshore wind recommendations and standards, which results in 33% probability for the event occurrence, making the risk very probable. On the other hand, Spanish ROM 0.0 recommendation [9] for the same or similar lifetime, and considered these structures as instantaneous destruction implies more than 200 years of return period, value higher than 50 years.
- There are many similarities between all scour problems, the bed shear stress causes scour regardless of whether the flow is wave-alone or current-alone, or the combined wave-current situation, but many different approaches have been adopted in each one. Up to now, formulations for the design of scour protection systems do not take into account most of terrain properties. Nowadays, there are not any formulations to characterize this phenomenon under combined wave and current condition. It is important to promote the use of formulations that can express the extent of scour protections as a function of waves due to the importance of climatic variables.
- It is important to establish clearly in offshore wind recommendations the regimen (Morison, Froude–Krilov or diffraction) to consider for wave loads determination. Also, it would be very helpful to give more information and support for the use in Froude–Krilov and diffraction regimens, because although Morison is habitually used for slender structures like monopiles, jackets or tripods, Froude–Krilovs and diffraction regimens are to be considered for GBS structures. Other important issue to be analyzed is if Morison et al. formulation can be applied for big pile diameter, more than 5 m.
- Lineal theory is rarely the most suitable wave theory but it is the theory considered in most equations. It can be checked with general project data introduced on Lè Mèhautè diagram that the most common wave theories are Stokes and Cnoidal. This can imply some uncertainties in the structural design. Furthermore, it is important to select the period to be used to calculate the wave length in the location of the maritime structure to be designed.
- Wave variables selection is important. The wave height assumed can be the significant wave height (H_s) or the maximum wave height (H_{max}). And the wave period is not the intrinsic one according DNV standard; the right one is the most stable in statistical or in spectral terms: the medium period (T_m or T_{02}).
- Up to now, typical piles used in maritime engineering have a maximum diameter around 2 m. On the other hand, monopiles used in offshore wind facilities, have a diameter around 5 m or even bigger diameters. The difference in piles size is evident, and this shall be taken into account because this can be risky. Changes in scale can imply failures and problems, as have previously happened in maritime engineering.
- DNV indicates that liquefaction has to be taken into account in seismic areas, but this has to be considered also during the pile driving and even during the operation phase because the dynamic loads.

As a result of this, there are a lot of improvements to make in this field. Some of them are mentioned in this paper, but the future experience in the matter will make it possible to detect more issues to be solved and improved.

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